PILE FOUNDATIONS FOR RIVER BRIDGES ACCORDING TO EC 7- 1

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ABSTRACT: Eurocode 7 Part 1 was published in Austria as ÖNORM EN 1997-1 already on May 1st, 2005, once more in an improved German language version on January 1st, 2006. On May 15th, 2009 a consolidated version was publishes including Corrigendum (AC) which was sent out by CEN in February 2009.
The National Annex (NA) to EC 7-1, called ÖNORM B 1997-1-1:2007-11-01, gives guidance how to apply EC 7-1 in Austria. For the design of pile foundations an additional standard ÖNORM B1997-1-3 is now being prepared by the responsible national committee and shall be published soon.
The coexistence period of old geotechnical standards based on a global safety system parallel to Eurocodes ended in Austria by June 2009. Therefore specific case histories of projects designed according to Eurocodes, especially to EC 7-1 are not yet available.
The authors of this paper were dealing with several river bridge foundations using large diameter bored piles during the last decades. So they try to show the difference between the design of such pile foundations based on the tradition in Austria, especially using often only a SLS design, and the new approach according to EC 7-1 by means of examples of Danube River bridges in eastern Austria already built several years ago, of one such bridge construction which is now under execution and of one river bridge project abroad in south-eastern Europe, close to the Danube River.
The types of pile foundations to be presented in this paper are different as follows: single piles combined to pile yokes, pile groups as usual and box-shaped pile groups for bridge piers in the river-bed.
Concerning box-shaped pile foundations it will be shown that additional definitions which are neither given in EC 7-1 nor in its NA for Austria are necessary for the design.
Finally some data on the behaviour of these pile foundations shall be presented.

1. Introduction

The National Annex to EC 7-1 for Austria, ÖNORM B1997-1-1 determines that for the design of spread foundations and also of pile foundations Design Approach 2 (DA 2 resp. DA 2*) has to be applied, that means that the effects of actions E_k are to be multiplied by partial factors γ_E and that the resistances R_k are to be divided by partial factors γ_R to get the design values E_d and R_d. More details to the Austrian national specifications are given in [1].
According to a decision of the national supervising committee in the Austrian Standards Institute which is responsible for the implementation of all Eurocodes the coexistence period with national standards, based on the deterministic global safety system, ended by June 2009. Starting from this date new projects, especially such of infrastructure and of governments are to be designed in Austria only according to Eurocodes. Also the Vienna building authority requires such a design since mid of 2009. Thus experience of projects already executed designed with partial factors according to Eurocodes is not yet available.

As the values of partial factors were determined for Austria in such a way – very similar to those determined by DIN in Germany – that the safety level of spread foundations and that of pile foundations should maintain as before comparison of design results of project of the last years based on the old global safety system with the results design calculations based on the partial factors theory is possible. Moreover it shall be shown that special types of pile foundations cannot be designed according to the EC 7-1 concept without additional regulations. To demonstrate these problems some different concepts of pile foundations of existing or former bridges across the Danube River in the eastern part of Austria and also a river bridge project in south-eastern Europe shall be presented here with comparable design result.

2. Short historic overall view

Piers of Danube River bridges in Austria, built in the second half of the 19th and in the first half of the 20th century usually had deep foundations using compressed-air-caissons. Also the last bridge that was built in Vienna in this period, the "Reichsbrücke", constructed a few years before the senior author of this paper was born, a special type of a suspension bridge, had such a foundation, based on an expertise concerning the subsoil conditions by Prof. Karl v. Terzaghi [2]. Unfortunately this bridge failed in the early morning of Sunday, August 1st, 1976 but not because of a failure of the foundation.
It was necessary to connect the city centre of Vienna with those parts of Vienna situated across the river as soon as possible by 2 temporary emergency bridges, one for the tram and one for the public traffic using prefabricated steel frame work parts each with a span of about 80 m for the bridge construction itself and piers (yokes) consisting of large bored piles with 1,50 m diameter connected with reinforced concrete head beams. The senior author of this paper presented a report on the design, execution, load testing and behaviour of these temporary bridges at the 5th Danube-European Conference SMFE in Bratislava in September 1977 [3].
Starting in 1978 a new "Reichsbrücke" was built in Vienna with a bridge pier in the river-bed using a box-shaped deep foundation of large diameter bored piles. Prof. Heinz Brandl reported about this solution at several occasions [4], [5]. Meanwhile this type of pile foundation is quite usual for large bridge piers situated in the river-bed of the Danube and has become almost a standard solution. Meanwhile several bridges were constructed in this manner.
Actually another Danube River bridge "Donaubrücke Traismauer" is under execution some 60 km upstream of Vienna to close a gap within the outer highway ring around Vienna. Both bridge piers in the river-bed were founded in 2008 on such box-shaped bored pile groups (see Fig. 16 in section 5.3.4 of this paper) however with some special modifications concerning the design and the execution of the shafts of these piers [6], [7]. Also all other parts of this bridge with a total length of about 1 km are founded on bored pile of 1,20 m diameter however using pile groups as usual with pile execution also during 2008.

3. Determination of the resistance of large diameter bored piles

As it is known EC 7-1 gives several options how to determine the resistance of piles, especially of axially loaded large diameter bored piles in compression, e.g.:

1) by static pile load tests (clause 7.6.2.2),
2) by dynamic pile load tests (clause 7.6.2.4),
3) by deriving values from soil investigation results, based on pile load test results and on experience (clause 7.6.2.3)

and by other methods.

Method 1) is applied in Austria only in special cases with problematic subsoil conditions such as soft or very loose and highly compressible soils. For typical soil conditions in the Danube region in Lower Austria and in Vienna several pile load test results exist since the seventies of the last century. These tests were performed on bored piles of usually 0,90 m diameter.

Method 2) was applied in Austria only in a few cases. Therefore this method will not be further discussed.
Method 3) was widely used in Austria up to now in many cases where soil conditions were clear and well known. This method is in accordance with a national standard for pile design.
Tables containing permissible values of skin friction and base resistance of bored piles with diameters of at least 0.90 m are given in this national standard ÖNORM B 4440:1984. A revised version was published in 2001 considering the European standard EN 1536 for the execution of bored piles. This standard and its design method for bored piles however is not in accordance with the requirements of EC 7-1 with the partial factor safety system and a design both for ultimate limit state (ULS) and for serviceability limit state (SLS).
Consequently a new national standard ÖNORM B 1997-1-3 is now being prepared and shall be published soon following these requirements of EC 7-1. This new standard shall be applicable for all types of piles (not only bored piles) and shall contain an informative annex with tables of characteristic values of skin resistance and base resistance for the design of bored piles. To be able to apply this design method in practice it is necessary to define the properties density, relative density of coarse grained soils by results of SPT and/or by dynamic probing (DPH or DPM) and those of fine grained and cohesive soils by values of I_c, c_u and q_u.
As cone penetration and pressiometer testing is rather rare and therefore experience on correlations with pile resistance is rather poor in Austria such parameters are not yet introduced into the tables of this draft standard.
Fig. 1 and 2 contain originals of the draft tables no. C.4 to C.7 of the national draft standard mentioned above containing characteristic values of base resistance of fain grained cohesive soils depending on related pile head displacements *s/D* and of skin resistance both for ULS- and SLS design approach.

Tabelle C.4 — Charakteristische Pfahlsohlwiderstände ($q_{b;k}$) in grobkörnigen (nicht-bindigen) Böden, abhängig von N_{30}-Werten, ohne Fußverpressung

bezogene Pfahlkopfsetzung s/D_b	Charakteristische Pfahlsohlwiderstände ($q_{b;k}$) in weitgestuften Sanden und Sand-Kies-Gemischen		
	mitteldicht[1])	dicht[2])	sehr dicht[3])
	MN/m²	MN/m²	MN/m²
0,005	0,30	0,40	0,50
0,01	0,55	0,80	1,00
0,02	1,05	1,40	1,75
0,03	1,35	1,80	2,25
0,05	1,90	2,50	2,95
0,075	2,50	3,10	3,55
0,10 (= s_g/D_b)	3,00	3,50	4,00

[1]) N_{30}-Wert ≥ 10
[2]) N_{30}-Wert ≥ 30
[3]) N_{30}-Wert ≥ 50
Zwischenwerte dürfen geradlinig interpoliert werden.

Tabelle C.5 — Charakteristische Pfahlsohlwiderstände ($q_{b;k}$) in feinkörnigen (bindigen) Böden, abhängig von der Konsistenzzahl I_c, ohne Fußverpressung

bezogene Pfahlkopfsetzung s/D_b	Charakteristische Pfahlsohlwiderstände ($q_{b;k}$) in Schluffen, tonigen Schluffen und Tonen		
	steif[1])	sehr steif[2])	halbfest[3])
	MN/m²	MN/m²	MN/m²
0,005	0,10	0,15	0,25
0,01	0,15	0,30	0,45
0,02	0,35	0,60	0,90
0,03	0,45	0,80	1,15
0,05	0,60	1,10	1,60
0,075	0,70	1,40	2,00
0,10 (= s_g/D_b)	0,80	1,50	2,20

[1]) $I_c > 0,75$
[2]) $I_c \geq 0,90$
[3]) $I_c > 1,00$
Zwischenwerte dürfen geradlinig interpoliert werden.

Fig. 1. Draft tables no. C.4 and C.5 of pr ÖNORM B 1997-1-3

Tabelle C.6 — Charakteristische Pfahlmantelwiderstände ($q_{s;k}$) in grobkörnigen (nicht-bindigen) Böden, abhängig von N_{30}-Werten, ohne Mantelverpressung

Grobkörnige Böden		Charakteristischer Wert des Pfahlmantelwiderstandes ($q_{s;k}$)	
		für den Gebrauchstauglichkeits-nachweis	für den Grenztragfähigkeits-nachweis
Lagerungsdichte	N_{30}	MN/m²	MN/m²
locker	4	0,030	0,045
mitteldicht	10	0,050	0,075
	20	0,060	0,090
dicht	30	0,070	0,105
	40	0,095	0,142
sehr dicht	≥ 50	0,120	0,180
Zwischenwerte dürfen geradlinig interpoliert werden.			

Tabelle C.7 — Charakteristische Pfahlmantelwiderstände ($q_{s;k}$) in feinkörnigen (bindigen) Böden, abhängig von der Konsistenzzahl bzw. einachsigen Druckfestigkeit q_u, ohne Mantelverpressung

Feinkörnige Böden		Charakteristischer Wert des Pfahlmantelwiderstandes ($q_{s;k}$)	
Konsistenz	Druckfestigkeit q_u	für den Gebrauchstauglichkeits-nachweis	für den Grenztragfähigkeits-nachweis
	MN/m²	MN/m²	MN/m²
weich[1])	0,03	0,010	0,015
steif[2])	0,06	0,020	0,030
	0,10	0,035	0,052
halbfest[3])	0,13	0,045	0,067
	0,16	0,055	0,082
fest[4])	≥ 0,20	0,070	0,105

[1]) $I_c \leq 0{,}75$
[2]) $I_c > 0{,}75$
[3]) $I_c > 1{,}00$
[4]) $I_c > 1{,}25$
Zwischenwerte dürfen geradlinig interpoliert werden.

Fig. 2. Draft Tables C.6 and C.7 of pr ÖNORM B1997-1-3

The values of these tables shown above have to be applied on the basis of the national specifications concerning EC 7-1 (ÖNORM EN 1997-1) and national supplements, given in the Austrian NA (ÖNORM B1997-1-1) in its section 4.5 for pile foundations as follows:

- Design Approach 2 resp. 2* (clause 2.4.7.3.4.3 of EC 7-1) has to be applied as already mentioned in the introduction of this paper with the combination of partial factors *A1* "+" *M1* "+" *R2.*
- The partial factors specified for Austria, which are taken unchanged from the tables of Annex a of EC 7-1 are given in the tables no. 5 to 8 below:

Tabelle 5 — Teilsicherheitsbeiwerte für Beanspruchungen (γ_E)

Beanspruchung		Symbol	Wert		
Dauer	Bedingung		BS 1	BS 2	BS 3
ständig	ungünstig	γ_G	1,35	1,20	1,00
	günstig	γ_G	1,00	1,00	1,00
veränderlich	ungünstig	γ_Q	1,50	1,30	1,00
	günstig	γ_Q	0	0	0

Fig. 3. Table no. 5 from ÖNORM B 1997-1-1 (NA to EC 7-1 for Austria)

- As it can be seen the partial factors for the effects of actions are subdivided according to the tradition in Austria into three design situations ("Bemessungssituationen") BS 1

(persistent), BS 2 (transient) and BS 3 (accidental) just like the way used by DIN in Germany.

Tabelle 6 — Teilsicherheitsbeiwerte für Bodenkenngrößen (γ_M)

Bodenkenngröße	Symbol	Wert
effektiver Reibungswinkel	$\gamma_{\varphi'}$	1,00
effektive Kohäsion	$\gamma_{c'}$	1,00
undrainierte Scherfestigkeit	γ_{cu}	1,00
einaxiale Druckfestigkeit	γ_{qu}	1,00
Wichte	γ_{γ}	1,00
[a] Dieser Beiwert wird auf tanφ' angewendet.		

Tabelle 7 — Teilsicherheitsbeiwerte für die Widerstände von Rammpfählen, Bohrpfählen und Schneckenbohrpfählen (γ_R) für alle Bemessungssituationen

Widerstand	Symbol	Wert
Spitzendruck	γ_b	1,10
Mantelreibung (bei Druck)	γ_s	1,10
Gesamtwiderstand (bei Druck)	γ_t	1,10
Mantelreibung bei Zug	$\gamma_{s;t}$	1,15

Tabelle 8 — Streuungsfaktoren ξ zur Ableitung charakteristischer Werte aus statischen Pfahlprobebelastungen für alle Bemessungssituationen

ξ für n =	1	2	3	4	≥ 5
ξ_1	1,40	1,30	1,20	1,10	1,00
ξ_2	1,40	1,20	1,05	1,00	1,00
n Anzahl der probebelasteten Pfähle					

Fig. 4. Tables no. 6 to 8 of ÖNORM B 1997-1-1 (NA to EC 7-1 for Austria)

- Table no. 7 contains partial factors on the pile resistance for all types of piles and is a summary of the tables no. A.6 to A.8 of Annex A of EC 7-1 were all partial factors in column *R2* are the same, that is γ = 1.10 for piles in compression and γ = 1.15 for pile in tension.
- Table no. 8 is identical with table no. A.9 of Annex A of EC 7-1 with correlation factors for pile resistance derived from the results of static pile load tests.
- However table no. A.10 of Annex A of EC 7-1 with correlation factors ξ_3 und ξ_4 are not applicable in Austria because of a lack of experience in cone penetration testing. Therefore it was necessary of close the gap of safety by a model factor according to EC 7-1, which was determined to $\eta_{P;c}$ = 1.30.

4. Typical subsoil conditions in the Danube region of eastern Austria

The subsoil conditions in the Danube region of eastern Austria and of course also in the lowland of the Danube River in Vienna on both sides are characterized by the geological period of tertiary transgressions with marine deposits of silts, clay and fine sands, very often in intermediate layers. These deposits are covered by coarse grained sediments of the quarternary period, mostly sands and gravels and youngest soils of Holocene along the river consisting of silt and sand ("Auböden"). A typical soil profile is given in Table 1 showing four characteristic layers of different geological age.
The groundwater-table lies usually in layer (3) but naturally it is corresponding to the water level of the Danube River.

Table 1. Typical soil profile in the Danube region in eastern Austria

(1)	A	Organic topsoil and anthropogene fills, variable layer thickness
(2)	FSa, Si	Holocene alluvial deposits of Danube floods, consisting of sands and silt (“Auböden”) layer thickness usually up to 5 m
(3)	Gr, Sa	Partly alluvial and partly Pleistocene deposits of gravels and sands, some stones and cobbles near the base (“Donauschotter”), layer thickness up to 10 m
(4)	Si, Cl, FSa	Tertiary marine deposits (Pannon, “Wiener Tegel”) consisting of silts and clays with intermediate layers of fine to medium grained sands, usually containing groundwater with confined pressure, reaching down to the bedrock in greater depth

Only in a few cases of bridges across the Danube River in eastern Austria bedrock is relevant, e. g. the bridge close to Melk Monastery (“Donaubrücke Melk”) upstream of Wachau.
Usually the layers (1) and (2) are not taken into account when estimating the pile resistance. For the design of the foundation of a bridge pier in the river-bed of Danube these two layers (1) and (2) are missing totally and layer (3) is very often of minor influence because of minor thickness of this layer. Thus main interest was always paid to determine the soil properties of the tertiary deposits being highly responsible for the pile resistance and the behaviour of the foundations of such Danube River bridge piers.

5. Case studies

5.1 Temporary Danube River bridges in Vienna 1976

5.1.1 General

It seems to be evident that because of top priority to build temporary bridges after the failure of the “Reichsbrücke” on August 1st, 1976 as soon and as quickly as possible there was no time to perform soil investigations or pile load tests for the design of these essentially necessary temporary bridges. One such bridge was built for the tram, one for public traffic.
Thus pile resistance or pile capacity had to be estimated with very conservative assumptions to ensure a safety against failure as high as possible. Additionally it has to be considered for this historic case history that bored piles of 1,50 m diameter, executed in the river with the machinery on ships or pontoons had to be bored with permanent steel casings not only above the river-bed but also down into the ground to a certain depth. There was only poor experience on the resistance of such piles installed by borings with permanent steel casings in these soil conditions.

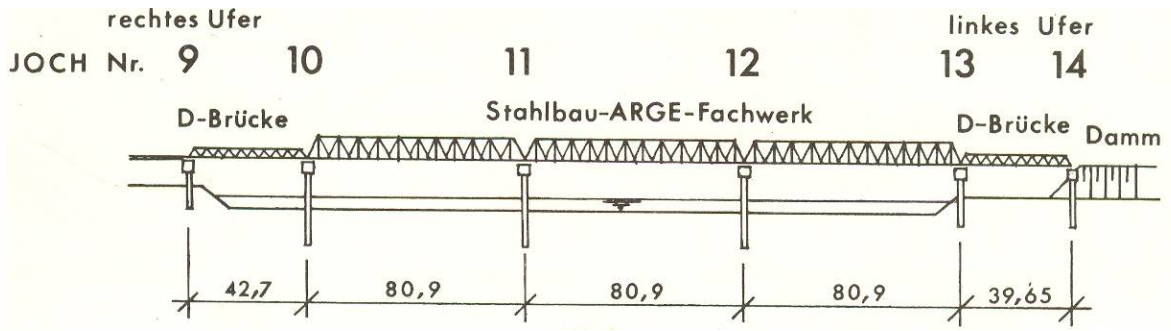

Fig. 5. General longitudinal layout of the emergency bridges [3]

This oldest example of our contribution deals with the bridge for the tram and shall give a comparison of the design of the pile foundation used in the year 1976 with a design

calculation according to EC 7-1, using the additional requirements for pile foundations of the draft national standard pr ÖNORM B 1997-1-3.
The bridge for the tram which was already finished on October 15th, 1976, that means only 75 days after the disaster, had 6 bridge piers as shown in Fig. 5 above, 3 of them in the river bed and 3 piers at the river banks. This bridge was a double bridge, each single bridge for one direction of the tram line. Each pier was formed by 4 bored piles of 1.50 m diameter. Every 2 piles of each pier ("yoke") were connected by pre-fabricated reinforced concrete beams (1.80 m x 1.80 m x 5.90 m) as shown in Fig. 6 below. The embedded length of the piles beneath the bottom of the river was 13 m.

The maximum working load for each single pile of the piers no. 11 and 12 was determined to

$$Q_{tot} = 1.73 \text{ MN (permanent)} + 0.97 \text{ MN (variable)} = 2.70 \text{ MN (total)}.$$

The bridge superstructure itself consisted of pre-fabricated steel frameworks of two different types, smaller units with a span of about 40 m of pioneers of the Austrian army ("Bundesheer") and larger units produced by Austrian steel contractors while foundation works were carried out.

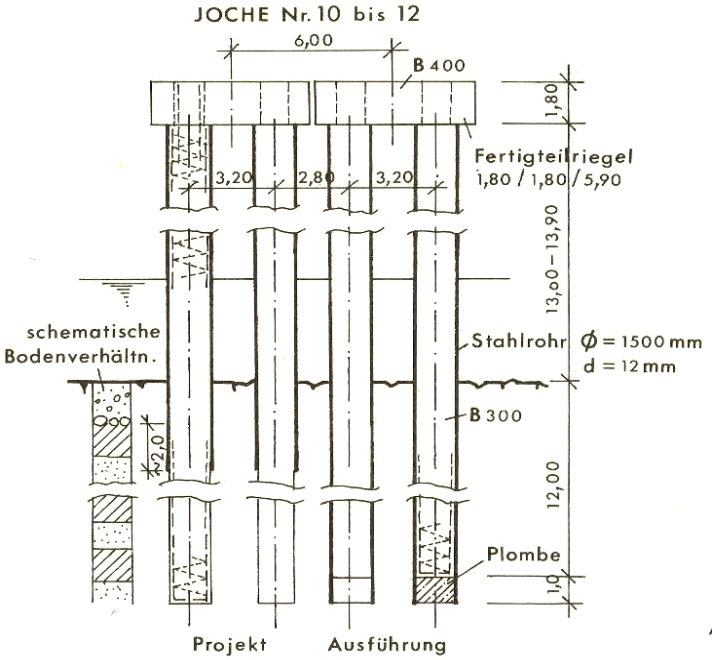

Fig. 6. Cross section of the piers for the tram bridges ("yokes" no. 10 to 12), [3]

As there was not time for soil investigation by borings, field and laboratory testing the design was based on an old, but very famous geotechnical expertise by the late Prof. Karl v. Terzaghi dated May 1933 for the project f a new bridge "Reichsbrücke" which was built between 1934 and 1937 and which failed 1976 only 40 years later. Fig. 7 below shows the geological profile of 1932 across the Danube River parallel to the predecessor bridge of Reichsbrücke, the "Kronprinz-Rudolf-Brücke", based on the results of 5 test borings.
The target concerning the displacements of the bridge piers given of the emergency bridges by the designer were 30 mm total settlement of the pile heads and a maximum differential settlement between 2 piles of 10 mm.

5.1.2 Pile design 1976

The estimation of the axial resistance of one single pile in compression was based on an ultimate base resistance similar to that of a spread foundation in a depth of 13 m below the bottom of the river according to the German standard at that time DIN 4017-1 and a very conservative calculated skin resistance [8] according to the formula

$$R_s = A_s * t_{red} \, (K_o . \sigma_m . \tan \delta + c')$$

with

A_s area of the pile skin per metre,

t_{red} reduced embedded length of the pile 8,00 m,

K_o earth pressure coefficient at rest,

σ_m average horizontal earth pressure around the pile,

δ angle of friction between soil and the pile skin

and with the following empirical soil parameters:

ϕ = 25° average angle of shear of the tertiary strata,

c' = 20 kPa effective cohesion of the tertiary strata,

γ = 11 kN/m³ unit weight of the tertiary strata.

Thus the ultimate pile resistance in compression resulted to R_{tot} = 5.86 MN, consisting of about 82 % of the base and only 18 % skin resistance and the global safety against failure was

$$\eta_q = 5.86/2.70 = 2.17 > 2.00.$$

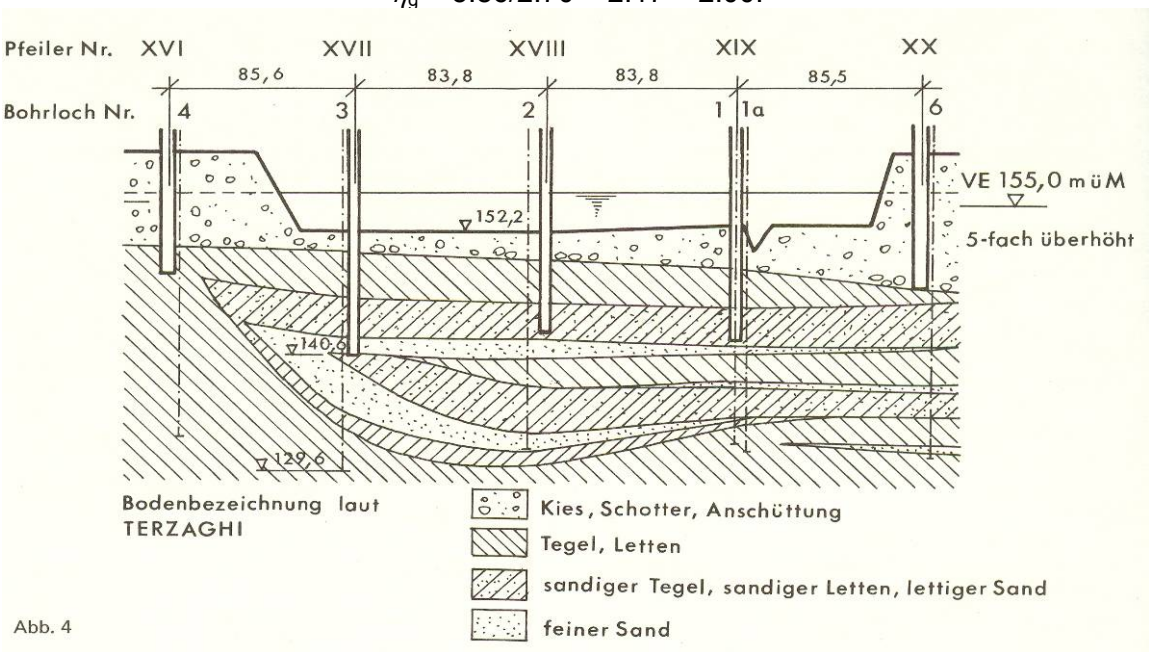

Fig. 7. Longitudinal geological profile (Terzaghi, 1932), [3]

To protect the bridge piers which only consisted of bored piles with permanent steel casings against effects of horizontal loads, especially against impact of ships, sheet pile walls were rammed around every pier of bored piles with a filling of rock ("Wasserbau-Wurfsteine").

5.1.3 Pile design according to EC 7-1

The estimation of the ultimate axial pile resistance of a single pile in compression (ULS design) according to EC 7-1 based on the values of tables of pr ÖNORM B1997-1-3 for an average consistency of the sandy and clayey silts of about I_c = 1.00 with

$q_{b;k}$ = 1.85 MN/m² (ultimate base resistance corresponding to s_g = 0.1* D_b from Table C.5)

$q_{s;k}$ = 0,06 MN/m² (ultimate skin resistance from Table C.7)

is [see EC 7-1, clause 7.6.2.2 (12) for piles in compression]

$$R_{c;k} = R_{b;k} + R_{s;k} =$$

$$1.767*1.85 + 0.06*4.712*8.0 = 3.269 + 2.262 = 5.53 \text{ MN},$$

consisting of about 59 % base and 41 skin resistance, a quite different relation than that of the design of 1976, but a more or less similar value of the total resistance.

With the partial factors $\gamma_b = \gamma_s = \gamma_t$ = 1.10 for piles in compression and a model factor according to EC 7-1, clause 2.4.1.(5) and clause 2.4.7.1 (6) $\eta_{P;c}$ = 1.30 the design resistance of a single pile is therefore

$$R_{c;d} = 5.531/1.1*1.3 = 3.86 \text{ MN}$$

The design value of the effects of actions for a single pile with $\gamma_G = 1.35$ and $\gamma_Q = 1.50$ is

$$E_d = 1.73*1.35 + 0.97*1.50 = 2.33 + 1.455 = 3.785 \text{ MN} < R_{c;d} = 3.86 \text{ MN}.$$

The serviceability limit state design (SLS design) also using relevant values from tables C.5 and C.7 of the draft national standard pr ÖNORM B 1997-1-3 with the same input of I_c = 1.00 and the tolerable pile settlement mentioned above in section 5.1.1 of s = 30 mm gives $q_{b;k}$ = 0.75 MN/m² (base resistance) and $q_{s;k}$ = 0.04 MN/m² (skin resistance). Introducing all partial factors γ = 1.00 (including the model factor mentioned above) the design resistance results to

$$C_d = 1.767*0.75 + 0.04* 4.712*8.0 = 2.83 \text{ MN} > E_d = 2.70 \text{ MN},$$

with about 47 % base and 53 % skin resistance.

Both the ULS design and the SLS design calculation show satisfactory correspondence with the design results of 1976. However the relations between base and skin resistance is quite different for these different methods of pile design.

5.1.4 Settlement behaviour of the emergency bridges

These temporary emergency bridges were built only for a life time of less than 5 years, but were constructed in minimum time of execution work.
Immediately after completion of the temporary bridge for the tram full scale static and dynamic load tests yoke by yoke were performed using a train with ballast equal to the calculated variable temporary loading of this bridge. These testings of every pile yoke were successful and showed pile settlements of 10 mm to 15 mm, i.e. 1 % of the pile diameter or only 10 % of the pile head displacement which usually defines the ultimate pile resistance as stated in EC 7-1, clause 7.6.1.1 (3). The differential settlements measured during the load tests were less than 3 mm (see Fig. 8 below).

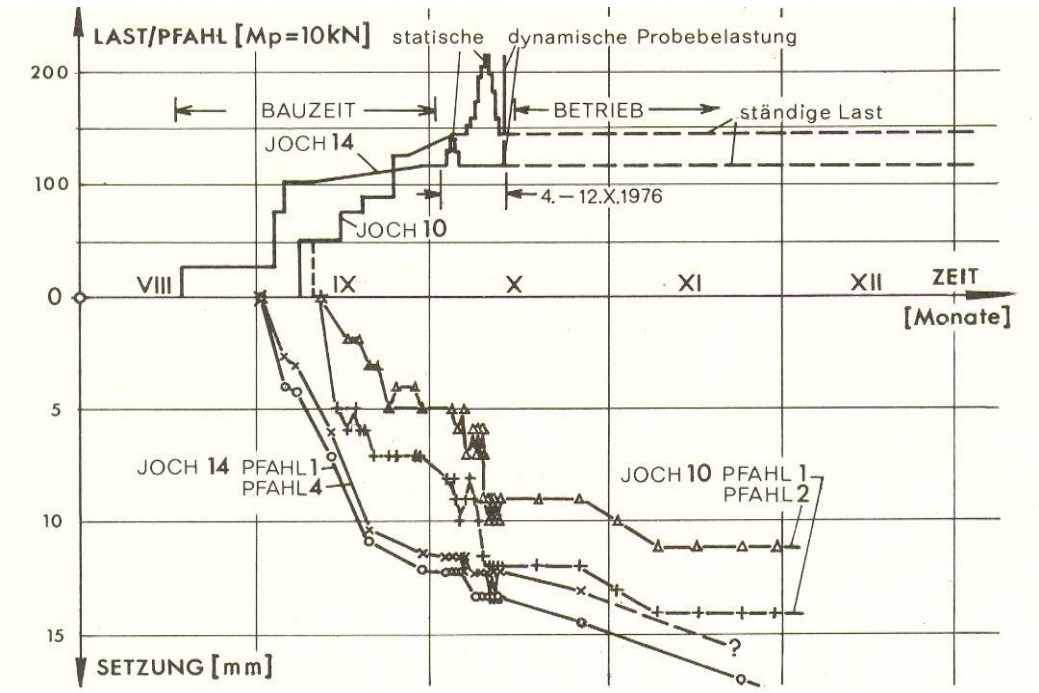

Fig. 8. Results of the static and dynamic test loading of the emergency bridge for the tram [3]

5.2 Danube River Bridge Pöchlarn - box foundation (BF)

5.2.1 Design model and verification concept for box foundations (BF)

5.2.1.1 Design Model

Consequently, in principle, two different safety verifications need to be performed when looking at limit states in BFs (Fig. 9*)*:

- the theoretical base failure strength of the single piles;
- the base failure strength of the box as a quasi-monolith.

Failure analysis of a single pile

This model assumes theoretical failure of a single pile, with the base failure pattern located predominantly outside the box. However, as when calculating soil failure resistance in accordance with B 4432 (old), the partial safety factors for the resistances can be reduced compared with ÖNORM B 4435-2 and ÖNORM B 1997-1 (see suggestion in table 3), skin friction only being factored in at the outer and inner surfaces of the BF. Skin friction is examined in accordance with chapter 3 of this paper.
The mutual influence of the base areas of the foundation elements is taken into account by considering the relationship between individual base areas and cell diameters, as a result of which the computed soil failure load may increase or decrease.

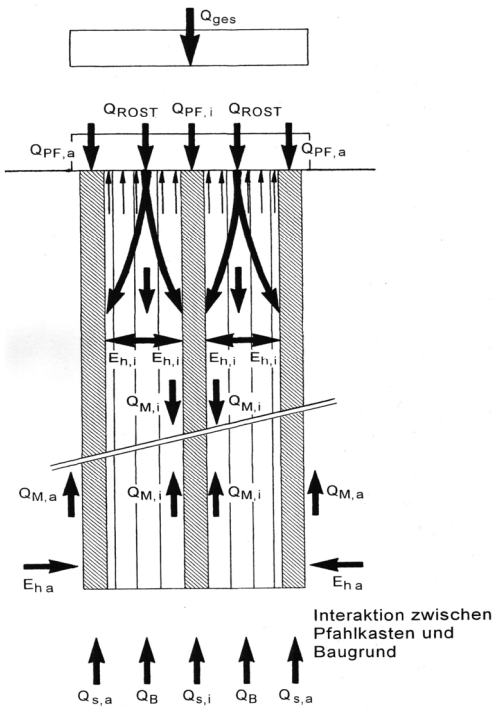

Fig. 9. Load distribution on the box, verification of the box as a quasi-monolith

For the purposes of this analysis, the computed soil failure load calculated according to ÖNORM B 4435–2 (characteristic soil parameters) is determined for a theoretical “shallow foundation” foundation. Table 2 contains a proposal for the application of partial factors. For this verification, skin friction can only be factored in at the outer wall of the box. Skin friction is examined in accordance with chapter 3 of this paper.

5.2.1.2 Verification of the geotechnical limit states (GEO) for box foundations

[9], [10] and [11] assume the usual global safety factors for soil failure analyses applying the global safety concept contained in ÖNORM B 4432 (old). They are η_1 = 3.5 or 2.75 when analyzing a monolithic box (standard loading case, extreme loading case) and η_2 = 1.5/1.2 when analysis an individual pile.

The internal and external structural stability of a box foundation must now be verified in accordance with Eurocode 7-1 and taking into account ÖNORMEN B 1997-1-3, ÖNORM B 1997-1, ÖNORM EN 1997-1 and ÖNORM B 4435-2 using design approach 2.

The measurement process applies design approach 2 in accordance with EN 1997-1, 2.4.7.3.4.3(1)P, the partial factors being applicable to the effects of actions and to the soil resistances.

The basic action (Q_{Ek}) and soil failure resistance (Q_{Rk}) are initially determined at a characteristic level. The action variable is then increased by the partial factor for actions and resistance is reduced by the partial factor for GEO and soil failure in accordance with table 3. The characteristic soil failure resistance ($Q_{R,k}$) is calculated in accordance with ÖNORM B 4435-2:1999, 6.
The design value for the basic action (Q_{Ed}) is obtained from the design values of the vertical and horizontal force components of the actions. Soil failure resistance is determined applying characteristic values. The computed soil failure resistance is reduced by the partial factors and the model factors.

Analysis is carried out on the basis of the following criteria:

$$E_d \leq R_d \quad \text{and} \quad Q_{E,d} \leq Q_{R,d}$$

Table 2. Suggested partial factors for box foundations (skin friction) – Design Approach 2 pursuant EC 7-1

Partial Safety Factors for Box Foundations (Skin Friction)									
Design approach 2 pursuant to EN 1997-1, 2.4.7.3.4 Geotechnical Limit State (GEO)	Effect of Actions (γ_E)				Soil Parameter (γ_M)			Resistance of BF (γ_R) (Skin Friction)	
	A1				**M1**			**R2**	
	Permanent unfavorable	Permanent favorable	Variable favorable	Variable favorable	Effective friction angle	Effective cohesion	Unit weight	Skin friction (compression)	Skin friction (tension)
	γ_G	γ_G	γ_Q	γ_Q	$\gamma_{\varphi'}$	$\gamma_{c'}$	$\gamma_{\gamma'}$	γ_R	γ_R
Design situation DS 1	1,35	1,0	1,5	0	1,0	1,0	1,0	1,1	1,15
Design situation DS 2	1,20	1,0	1,3	0	1,0	1,0	1,0	1,1	1,15
Design situation DS 3	1,0	1,0	1,0	0	1,0	1,0	1,0	1,1	1,15

5.2.1.3 Verifying the serviceability of box foundations

According to Eurocode 7, the serviceability of box foundations must also be verified. This verification must be performed applying the characteristic soil parameters and the characteristic actions. This means that all partial factors are set at 1.0. The deformation state of the entire system is determined on the basis of these parameters and actions.

Table 3. Suggested partial factors for box foundations (base resistance) – Design Approach 2 pursuant EC 7-1

Partial Safety Factors for Box Foundations (Base Failure)									
Design approach 2 pursuant to EN 1997-1, 2.4.7.3.4 Geotechnical Limit State (GEO)	effect of actions (γ_E)				Soil Parameter (γ_M)			Resistance of BF	Model Factor
	A1				***M1***			***R2***	
	Permanent unfavorable	Permanent favorable	Variable favorable	Variable favorable	Effective friction angle	Effective cohesion	Unit weight		
	γ_G	γ_G	γ_Q	γ_Q	$\gamma_{\varphi'}$	$\gamma_{c'}$	$\gamma_{\gamma'}$	γ_R	η
Verification as Quasi-monolith									
Design situation DS 1	1,35	1,0	1,5	0	1,0	1,0	1,0	1,65	1,5
Design situation DS 2	1,20	1,0	1,3	0	1,0	1,0	1,0	1,75	1,5
Design situation DS 3	1,0	1,0	1,0	0	1,0	1,0	1,0	1,85	1,5
Verification of Single Piles									
Design situation DS 1	1,35	1,0	1,5	0	1,0	1,0	1,0	1,1	1,0
Design situation DS 2	1,20	1,0	1,3	0	1,0	1,0	1,0	1,1	1,0
Design situation DS 3	1,0	1,0	1,0	0	1,0	1,0	1,0	1,2	1,0

The limit of serviceability in the ground is proven if the limit values for absorbable settlement ($s_{admissible}$) and settlement differentials ($\Delta s_{admissible}$) in the limit state for serviceability are not exceeded. The variables $s_{admissible}$ and $\Delta s_{admissible}$ are defined by the settlement sensitivity requirements for the structure.

$$s\,(Q) \leq s_{\mathbf{admissible}} \quad ; \quad \Delta s\,(Q) \leq \Delta s_{\mathbf{admissible}}$$

5.2.1.4 Box coefficients κ

The load bearing capacities of the various box types are described using the box coefficients κ (Fig. 10). The box coefficient states the proportion of the load that is transmitted from the "floor" to the soil. In the extreme case in which all loads are transferred through the soil, κ = 1.0. The limiting case κ = 0 means that all loads are transferred through the foundation elements.

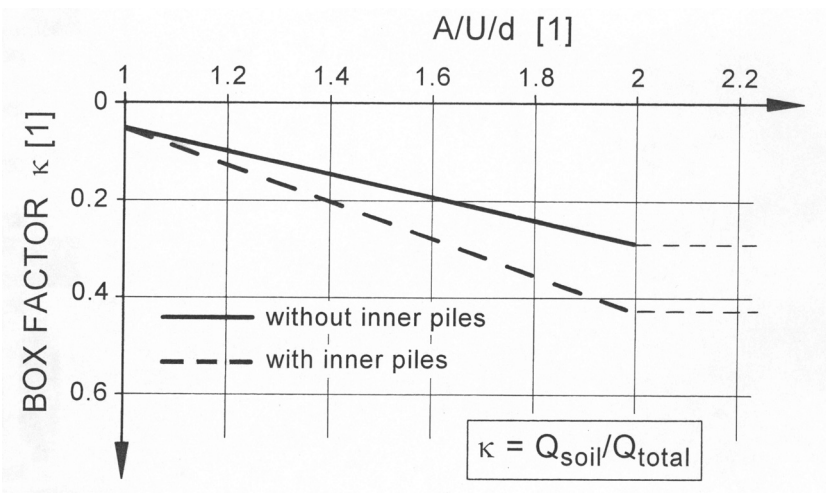

Fig. 10. Box coefficients κ

5.2.2 Geotechnical measurements of the box foundations (BFs) of Bridge Pöchlarn

5.2.2.1 General

Geotechnical measurements at river pier S1 (Fig. 11) of the Danube Bridge at Pöchlarn confirm the differences in load bearing performance between inner and outer piles and the part played by the raft and the soil enclosed by the piles in transferring the load into the subsoil. With the help of pile load measurements and pressure measurements [12] below the raft, the positive effect of high pressure soil stabilization (jet grouting) could be verified. It makes it possible to transfer the loads acting on the structure to the subsoil with much less deformation, as was confirmed by pile box settlement measurements during the structure's construction and after its completion.

5.2.2.2 Soil characteristics and the construction process

In the area of the river pier, we find "Melk" sands below quaternary sandy gravel with a thickness of roughly 4.0 to 6.5 meters. They consist of overconsolidated tertiary sediments in which slightly silty sands predominate but which also contain sandy coarse clay. As a rule, the soil's natural water content is below the plastic limit. In their consolidated and dehydrated state, Melk sands have a friction angle (φ) of between 31° and 36.5°.
The bored piles for the box foundations were created in July und August 1999. Then, high-pressure soil stabilization (jet grouting) was carried out inside the pile box down to roughly 7 meters below the underside of the raft. This implies embedment into the Melk sands to a depth of roughly 0.5 meters.

5.2.2.3 Results of the geotechnical measurements

Instrumentation of the box foundations of the Danube Bridge at Pöchlarn gave us further insight into the interaction between foundations and the ground. Instrumentation of one of the river piers took the form of concrete deformation measurements in selected piles at various levels, measurements of pile head loads and pile base pressures and measurements of concrete stress under the raft. These automated measurements took place during the construction phase and they are still being continued today.

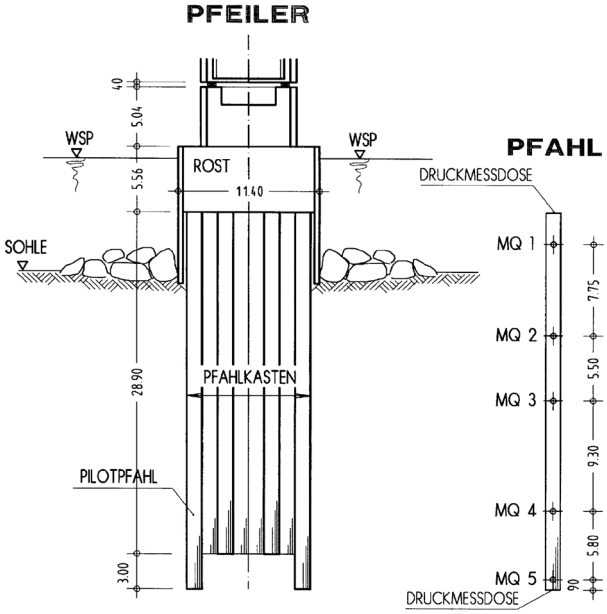

Fig. 11: Cross section of the pile box

In the upper part of the structure, some of the loads acting on the outer piles shifted to the inner piles roughly down as far as the river bed, depending on the distance between outer pile and inner pile. In the case of the three piles being measured, this was determined on the basis of the differences in compression in the first and second measurement cross sections. At the same time, the load on the inner piles in this area increased. As a result, in the second measurement cross section, the load on the piles located in the centre of the box reached a value of 125% of the average pile head load. Among other things, skin resistance was activated at the outer surfaces of the box and at the individual piles within the box. In the first eight meters below the river bed, skin friction at the outer surface of the box averaged 96 kN/m². Skin friction between measurement cross section MQ 3 and the base of the pile averaged 29 kN/m², which was 30% of the peak value. Compression was no longer registered at the lowest measuring level (Fig. 12).

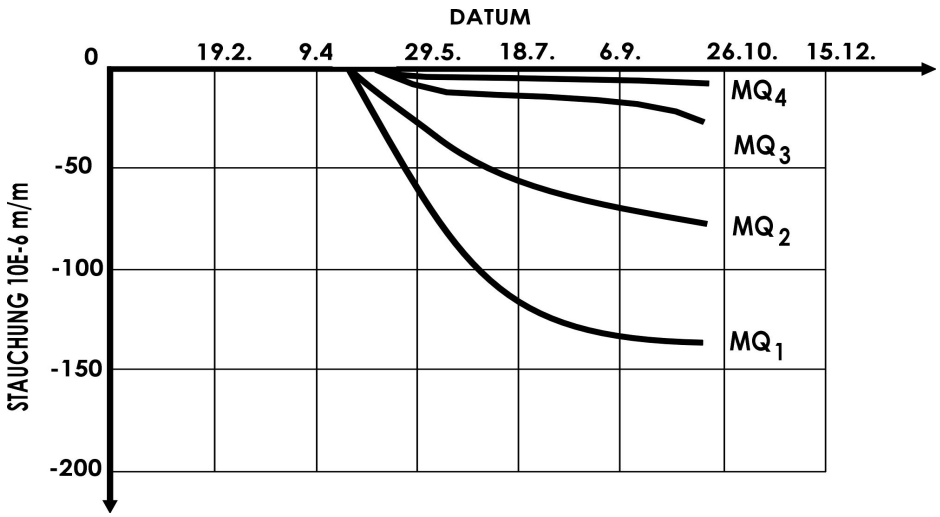

Fig. 12. Compression at measurement cross sections 1 – 5 (outer pile)

At the commencement of the structure's construction, the concrete compression load cells below the raft registered big increases in stress. Pressure on the ground rapidly reached roughly 500 kN/m² and soon peaked at 600 kN/m². After that, stress below the raft remained almost constant despite the increasing structural load whereas pile loads continued to increase (Fig 13). As a consequence, the proportion of the load on the raft came to 33% of the total load on the pile.

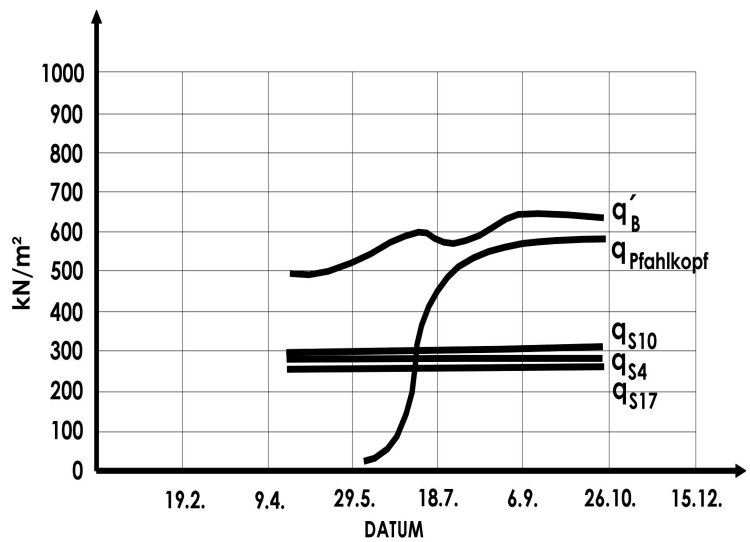

Fig. 13. Results of pressure measurements at the base of the pile and load cell measurements below the raft

5.3 Access bridges to Danube River Bridge Traismauer

5.3.1 General

According to special topographical conditions around the Danube River Bridge Traismauer some 60 km in the West of Vienna with woody areas for floods on both sides of the river a northern and a southern access bridges with lengths of about 450 m and 320 m were necessary. The distances of the bridge piers differ between 30 m und 40 m. The main bridge crossing the Danube River has a total length of approximately 350 m with spans of 100 m + 155 m + 100 m = 355 m total length. The width of the bridges (to parallel bridges for both directions of public traffic) is 31.40 m.
River Danube is dammed up here because of the hydro power station “Altenwörth” some 10 km downstream with a water depth of about 12 m.

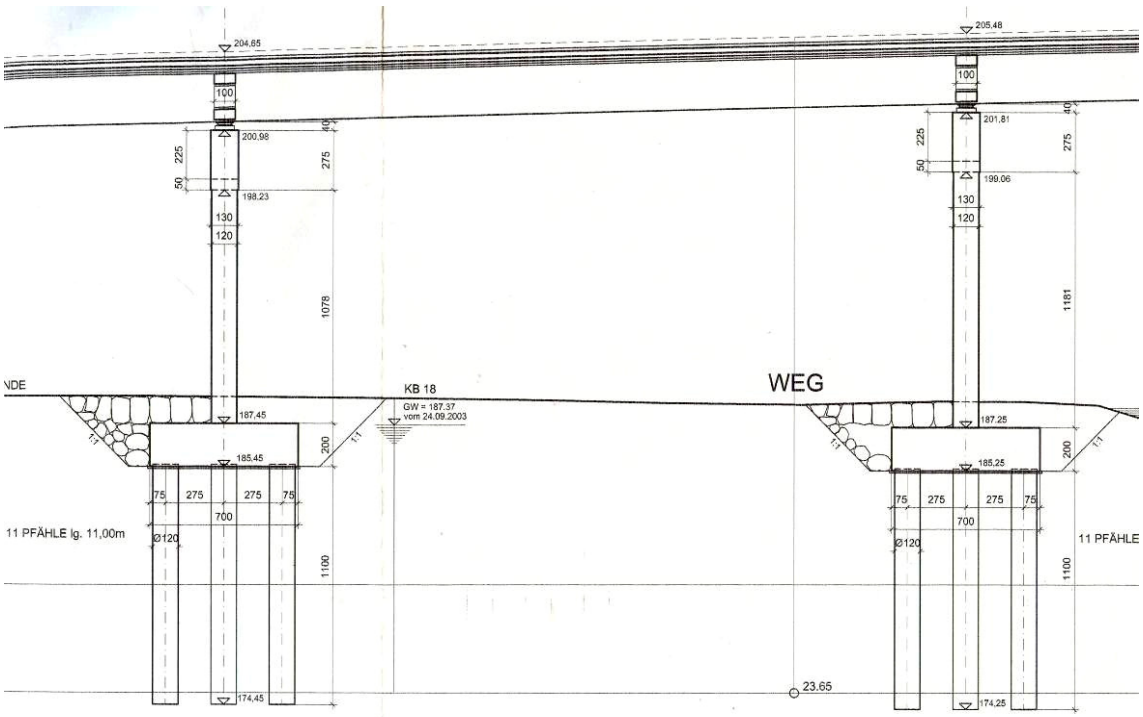

Fig. 14. Part of the vertical section plan of the southern access bridge Traismauer

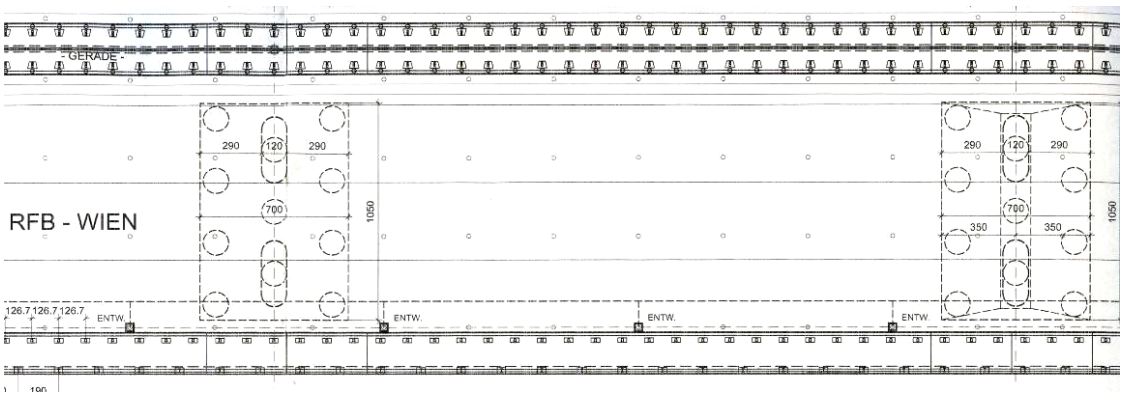

Fig. 15. Part of the ground plan of the southern access bridge

For the foundation of all piers and abutments of the access bridges conventional pile groups of bored piles of 1.20 m diameter and different lengths were applied. Pile heads are

connected by reinforced slabs. The drawings in Fig. 9 and 10 show an example of 2 piers of the southern access bridge.
The foundations of the 2 access bridges in the south and in the north consist of about 420 bored piles, with embedded lengths between 8.30 m and 10.20 m for the southern and between 10.30 m and 12.90 m for the northern bridges with a total length of about 4500 m. Taking the bored piles for all abutments and for the 2 main bridge piers in the river bed founded on box-shaped pile groups into account the total length of bored piles of 1.20 m diameter reaches more than 8500 m.

5.3.2 Preliminary pile design in the planning stage 2004/2007

This design of the pile foundations for all piers of the 2 access bridges south and north was based on the national standard for bored piles ÖNORM B 4440:2001-09.This standard only requires a serviceability limit state design introducing characteristic values of the permanent and variable actions or loads and empirical values of the pile resistance subdivided in the 4 tables no. 3 to 6 in this standard containing tolerable values of base resistance $\sigma_{s,zul}$ and skin friction $\tau_{M,zul}$.
The geological situation of Danube River Bridge Traismauer is typical as described in Table 1 in section 4 of this paper. Such typical soil profiles were found by several test borings, dynamic probings and laboratory testing and are summarized in Table 2:

Table 2. Typical soil profile for the southern access bridges

Layer no.	Type of soil	Thickness of layer
1	organic top soil	0 to 0.50 m
2	youngest deposits of silty fine sands	0.50 to 4.50 m
3.1	sandy gravels of medium density	5.00 m
3.2	sandy coarse gravels of higher density	3.50 m
4	tertiary silts and clays of medium to high plasticity, stiff to very stiff	from – 9.00 m down to very great depth

The next Table 3 shows the tolerable values of pile resistance from the national standard ÖNORM B 4440:2001-09, taken from the geotechnical expertise for this bridge project [9]:

Table 3. Tolerable values of pile resistance from an expertise [13]

Layer no.	Tolerable base resistance $\sigma_{s,zul}$ (MN/m²)	Tolerable skin friction $\tau_{M,zul}$ (MN/m²)
3.1	-	0.05
3.2	(0.80)	0.10
4	0.50	0.07

As it seems reasonable the layers 1 und 2 were not taken into account for the estimation of the pile resistance.
The tolerable axial resistance of a single pile in compression with 1,20 m diameter and an embedded length of 10,00 m in the layers 3.1, 3.2 and 4 according to Table 2 was thus determined to Q_{zul} = 2.93 MN, the allocated vertical displacement of the pile heads being less than 20 mm.
Depending on the length or slenderness of a pile the base resistance was reduced in this calculation by a factor α defined in the relevant national standard, with α = 0.49 for the given geometry of these piles. The results of this design resistance consist of approximately 90 % skin resistance and only 10 % base resistance which correspond with the usual practice in Austria and long term experience for similar soil conditions.
The design requirements for the pile groups contained minimum distances of pile axes of 3.00 m to derive the total design resistance of a pile group by multiplying the resistance of a single pile by the number of piles.

The very strict target concerning the displacements of the bridge piers given by the designer and the client was a long term differential settlement from one pier to the next of $\leq$ 5mm.

5.3.3 Pile design according to EC 7-1

Similar to the pile design method described in section 5.1.3 of this paper now it shall be shown how to estimate the axial resistance in compression of a single pile of a selected pier of the southern access bridge in Traismauer according to EC 7-1 and the Austrian national pile design method. The soil conditions are the same as given in the Table 3 in section 5.3.2 of this paper.
From the national draft standard pr ÖNORM B 1997-1-3, tables C.5 to C.7 we get the characteristic values of base and skin resistance as follows for the ULS design calculation:

Table 6. Values of pile resistance for the ULS design

Layer no.	Base resistance $q_{b;k}$ (MN/m²)	Skin resistance $q_{s;k}$ (MN/m²)
3.1	-	0.075
3.2	-	0.142
4	1.85	0.105

The total characteristic axial pile resistance in compression is then

$$R_{t;k} = 1.131*1.85 + 3.76\ (0.075*5.0 + 0.142*3.5 + 0.105*1.5) = 5.96 \text{ MN}.$$

To compare this result with that of the design according to previous ÖNORM B 4440:2001-09 all partial factors of EC 7-1 for design approach 2 (γ_G = 1.35 and γ_Q = 1.50 for the effects of actions, $\gamma_b = \gamma_s = \gamma_t$ = 1.10 for the pile resistance and the model factor $\eta_{P;c}$ = 1.30) are to be introduced with their resulting product similar to the former global factor of safety against failure η_G = 2.00.
This results to 5.96/2.0 = 2.98 MN, which is nearly the same as calculated according to ÖNORM B 4440:2001-09, but with a deviating relation of 65 % skin and 35 % base resistance.

For the SLS design calculation the tables C.5 to C.7 of pr ÖNORM B 1997-1-3 give the values of the pile resistance shown in Table 5 when presuming a pile head displacement of 20 mm (s/D_b = 0.02/1.20 = 0.0166):

Table 7. Values of pile resistance for the SLS design

Layer no.	Base resistance $q_{b;k}$ (MN/m²)	Skin resistance $q_{s;k}$ (MN/m²)
3.1	-	0.050
3.2	-	0.095
4	0.57	0.070

With all partial factors γ = 1.00 the total pile resistance is

$$C_d = 1.131*0.75 + 3.76\ (0.05*5.0 + 0.095*3.5 + 0.07*1.5) = 3.43 \text{ MN}$$

with a relation of 75 % skin and 25 % base resistance. It can be seen that the ULS design calculation is governing this pile design.

5.3.4 Settlement behaviour and concluding remarks

Monitoring of the displacements of the access bridges south and north which were built during 2008 and 2009 but are not yet opened for the public traffic showed settlements of only a few millimetres for each pier under the full permanent load which is more than 70 % of the total effects of all actions. So it can be expected that the long term settlements will be only about 50 % of the settlements and the differential settlements which were anticipated for the design.

By the end of the year 2009 also both superstructures of the main bridge across the Danube River were completed. At this stage of execution the vertical displacements of the 2 main bridge piers – both founded on box-shaped pile groups as shown in Fig. 11, each consisting of 48 bored piles of 1,20 m diameter with embedded lengths below the bottom of the river about 30 m – were measured to approximately 20 mm. This is nearly 80 % of the predicted long term settlement of 25 mm.

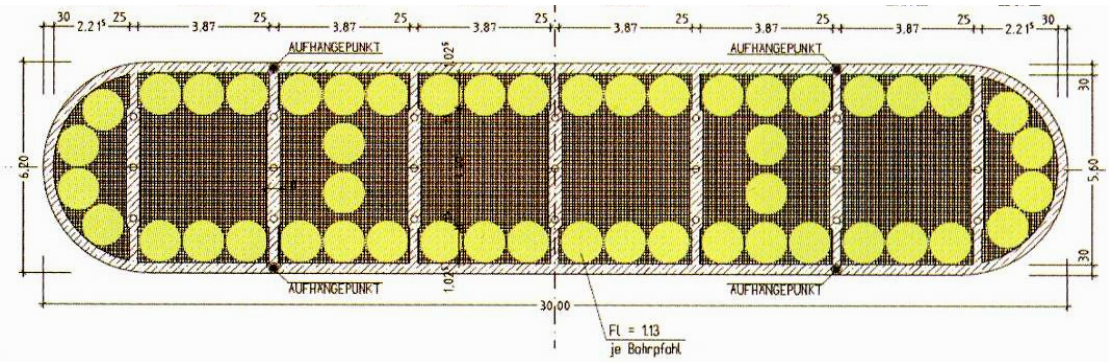

Fig. 16. Ground plan of the box-shaped pile foundation of the main bridge piers in the river-bed of Danube River Bridge Traismauer [7]

Concluding this example a few remarks shall be given to the very special and innovative construction method of the 2 river piers of the main bridge:
For each pier a ship-like pre-fabricated reinforced concrete element, 30 m long and 6 m wide, was brought by boat to its final position in the river. There it was lowered step by step to the bottom of the river adding further elements on top while filling it with unreinforced low strength concrete. After reaching the full height of the pier (from the bottom of the river to the level of the working platform, about 3 m above the surface of the river altogether 16 m) the 48 bored piles were executed with borings through this pier with a total depth of these borings of approximately 46 m. Thus the embedded length of the piles in the very stiff tertiary sediments of silts and clays (“Molasse”) is more than 25 m.

5.4 New bridge over River Sava in Belgrade

5.4.1 General

The bridge over the river Sava in Belgrade is the main structure on the inner semi-ring road from Omladinskih brigada Street to Paštrovićeva Street, and it crosses the river Sava in the zone of the downstream tip of Ada Ciganlija. The Sava Bridge is a 6 span, continuous superstructure with an overall length of 964 m between deck expansion joints. The main support system is a single pylon (pillar No. 6) asymmetric cable stayed structure with a main span of 376 m and a back span of 200 m. Four spans of 69 m, 108 m, 81 m and 81 m are continuous with the cable stayed section. The end span has a length of 50 m. The pylon is 200 m high. The overall deck width is 45.04 m and planned to carry 6 lanes of vehicular traffic, 2 tracks of a new light rail system and 2 lanes of a pedestrian/cycle way (see Fig. 17).

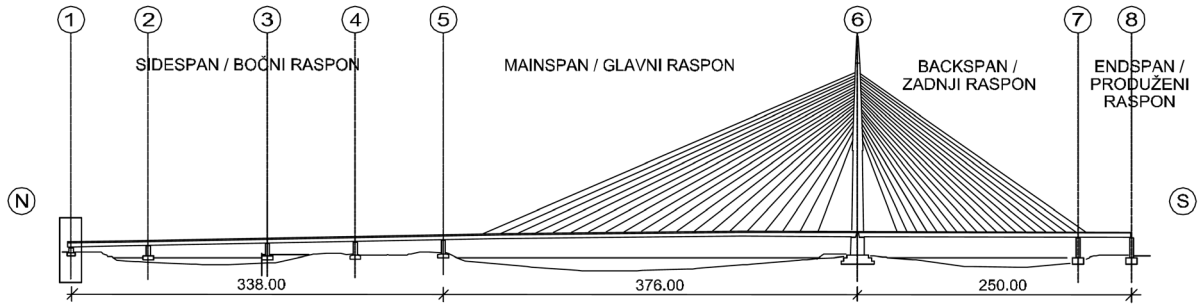

Fig. 17. Bridge over the River Sava. [14]

5.4.2 Ground properties

The following geological profile of the ground was recorded (see geological longitudinal section in Fig. 18):

- Layer n : Embankment as artificial surface cover
- Quaternary sediments:
  - Layer G-al: Silty-sandy clays with mud interbeds and lenses – facies of flood plain
  - Layer P-al: Medium-grained to fine-grained sands - river bed facies
  - Layer S-al: Gravels – River (fluvial)- lacustrine sediments
- Tertiary sediments:
  - Layer M: Weathered marly clays and marls and below Grey unaltered marls (not registered in boreholes around piers No. 7 and 8)
  - Layer R: Limestones, Sandstones
- Basic geological substratum:
  - Cretaceous sediments

At pier 6 ground water level is 1.48 m above surface in case of high navigation level (HNL) and subsoil is saturated. In case of low navigation level (LNL) the ground water level is in a depth of 2.18 m.

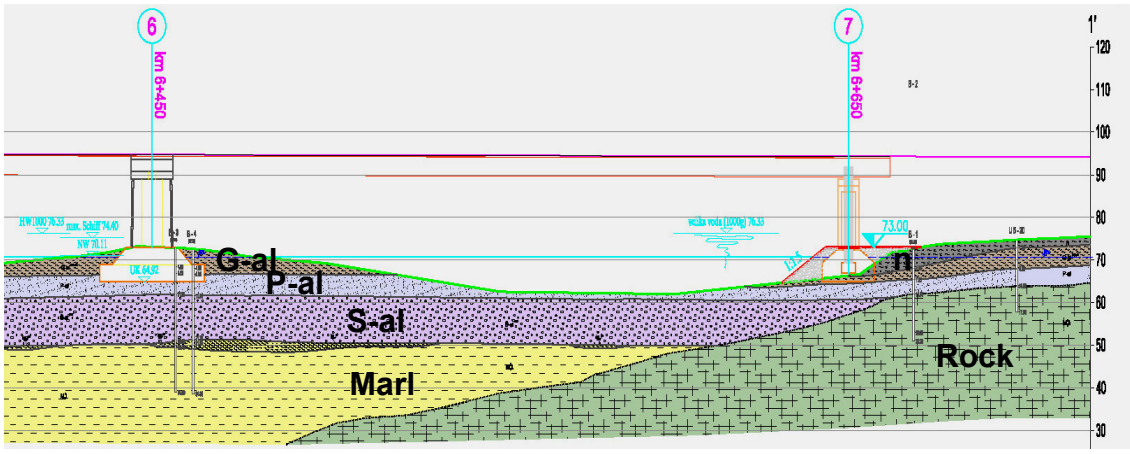

Fig. 18. Geological longitudinal section at piers 6 and 7 (not scaled). [15]

5.4.3 Foundation of piers, geotechnical and seismic design concepts

Due to investigated soil conditions and loads from bridge superstructure all piers are founded with piles. At piers 1 to 5 as well as 7 and 8 pile groups are designed. At pier 6 the

designed piles and diaphragm walls build up a box-shaped foundation, whereas the soil inside the box takes part at load transfer.

The foundation depth of piers 1 to 6 is within marl and the foundation depth of piers 7 and 8 within sandstone. At each pier the pile heads are connected by a pile cap for load transfer from the superstructure to the single piles.

Verification of vertical bearing capacity is performed according to EN 1997-1 (EC 7-1). The partial factors are assessed according to the national specifications concerning ÖNORM EN 1997-1 and national supplements. In the scope of the objective project no tension piles are performed so that an overall valid partial factor is assessed according to ÖNORM B 1997-1-1, Table 7:

$$\gamma_R = 1.10.$$

At piers 5, 6, and 7 pile load tests have been carried out. By means of the comparable stratigraphy of the ground at piers 1 to 6 the results of pile load tests at piers 5 and 6 can be transferred to piers 1 to 4 as well. Consequently, the following correlation factors are selected according to ÖNORM B 1997-1-1 Table 8:

$$\xi_1 = 1.40 \ (n = 1)$$
$$\xi_2 = 1.40 \ (n = 1)$$

At pier 6 a box-shaped foundation is designed. This foundation is composed of a compound body consisting of diaphragm walls and piles and the enclosed soil. This quasi-monolith transfers high vertical and horizontal forces. Piles and capping raft form a box, which acts physically like a “pot” turned upside down.

Consequently, the settlements are smaller than for conventional pile groups, and the earthquake resistance is significantly higher. Box-shaped foundations represent a special form of piled raft foundations utilising the enclosed soil core as an integrated load transfer member.

The following two calculation models are applied for the box-shaped foundation at pier 6:

- Calculation model A: Verification of vertical bearing capacity of single piles and diaphragm wall according to ÖNORM EN 1997-1 (EC 7-1). However, this standard includes no specifications related to the definition of shaft friction and base resistance pressure. Therefore, ÖNORM B 1997-1-3, which is currently established by the Austrian Standard Institute, is used. The new standard will replace ÖNORM B 4440 and will cover all pile types. Amongst others procedures for determination of characteristic values for ultimate limit state and serviceability limit state will be defined in tables including the base resistance pressure and shaft resistance (see tables above). The design value of pile resistance Rc;d for a bored pile at pressure is defined by:

  $$R_{c;d} = R_{c;k} / (\gamma_t * \eta_{P;c})$$

  with:

  - γ_t: partial factor of pile resistance; according to ÖNORM B 1997-1-1 Table 7: $\gamma_t = 1.10$.
  - $\eta_{P;c}$: scale factor (model factor) for axially loaded piles at pressure; according to ÖNORM B 1997-1-3 (draft) Table A.5 resp. ÖNORM B 1997-1-1: $\eta_{P;c} = 1.30$.
  - $R_{c;k}$: characteristic pile resistance as a result of shaft resistance and base resistance pressure; according to ÖNORM B 1997-1-3 (draft) Table C.4 to C.7.

  The characteristic values for shaft ꞩ and base resistance of piles in rock (limestone and sandstone at pier 7 and 8) are specified according to DIN 1054 – Appendix B, Table B.5 .
- Calculation model B: Quasi-monolithic: According to [16] a full bond effect between deep foundation elements and the closed soil is assumed. This compound body

comprises the outer circumference of the foundation if secant piles or diaphragm walls are installed. In the case of contiguous piles, the theoretical area should be reduced by at least half a pile diameter. For the quasi-monolith, only shaft friction along the outside surface of the foundation box may be taken into account.
The monolith-theory provides minimum pile or diaphragm wall loads. However, a full composite effect occurs only theoretically but hardly in practice. Therefore, relatively high safety factors are required. According to [16] calculations should be based on a global safety factor of $\eta \geq 3.0$, if conventional calculation methods for evaluating the base failure of equivalent "shallow" foundations are used. Finally, a global safety factor of ground failure of the raft foundation of $\eta = 3.5$ is used. Shaft friction is effective only around the outer perimeter of the box-shaped foundation and the base pressure is assumed to be effective over the entire base area of the monolith, whereas the base pressure does not exceed the actual overburden stress multiplied by the over consolidation ratio (OCR) of the marl in order to minimize foundation settlements and differential settlements. OCR was determined in the scope of laboratory tests in particular tests. Taking into account an average weight of soil of 20 kN/m² OCR ranges from 950 to 1940 kN/m².

For seismic design of foundations two approaches are applicable:
On the on hand dynamic soil parameters can be determined from geophysical field tests (e.g. cross-hole and down-hole tests) and/or dynamic laboratory tests (e.g. resonant columns test). Field tests deliver elastic data (dynamic shear modulus and dynamic elastic modulus) at small shear strains. However, increasing shear strains produce a reduction of the dynamic stiffness parameters. Depending on the seismic load and the corresponding shear strain the dynamic parameters can be identified. Stiffness matrix and damping matrix can be determined by using a suitable model, whereby radiation damping is represented by the damping matrix. However, radiation damping is always beneficial for the structure, thus neglecting this effect gives a solution on the safe side.
On the other hand seismic calculations can be performed by variation of the static stiffness if radiation damping can be neglected. In general, the quasi-static stiffness increases when dynamic loads are applied.
In the scope of the seismic design of the foundations of the Sava Bridge parametric studies were performed taking into account both approaches. Dynamic soil parameters were derived from geophysical tests and data given by the Serbian Seismic Institute in Belgrade.
A comparison of dynamic and quasi-static solutions yields that the dynamic approach overestimates the spring stiffness by means of the assumed rigid foundation behaviour during seismic load because the overall stiffness of the system is limited by the inherent rigidity of the foundation itself.
Thus, following parameters were chosen from the parametric studies:

- Lower limit: static value of horizontal and vertical modulus of subgrade reaction
- Upper limit: 10 times the static value of horizontal and vertical modulus of subgrade reaction

Characteristic values for shaft resistance and base resistance pressure need not to be modified from static values.
Consequently, in a case of an earthquake stiffening of the ground occurs, thus causing higher reaction forces but lower deformations. A softening of the ground can be excluded since the expected shear strains in the soil are small taking into account the magnitude of a presumed earthquake and the liquefaction potential of the layered ground is negligible.

5.4.4 Foundation of the pylon at pier No. 6

In the following the box-shaped-foundation of pier 6 is described as an example for pier foundations at Sava Bridge.

The foundation layout comprises the following parameters (see Fig. 19):

- piles: 113 piles with a diameter of 1.5 m
- pile length: 29.0 m
- diaphragm wall: thickness of 1.0m, length of 37 m
- pile cap: diameter of 25.0, 30.0 and 34.0 m and a total thickness of 8.0 m

The total load (serviceability limit state) amounts to 602 MN, whereas dead load of piles, diaphragm wall and pile cap are not included. The maximum working load of a single pile is 3.7 MN. Taking into account the additional load due to the dead load of the pile cap (uplift is considered for ground water level at low navigation level), the maximum working load of a single pile amounts to 4.4 MN.

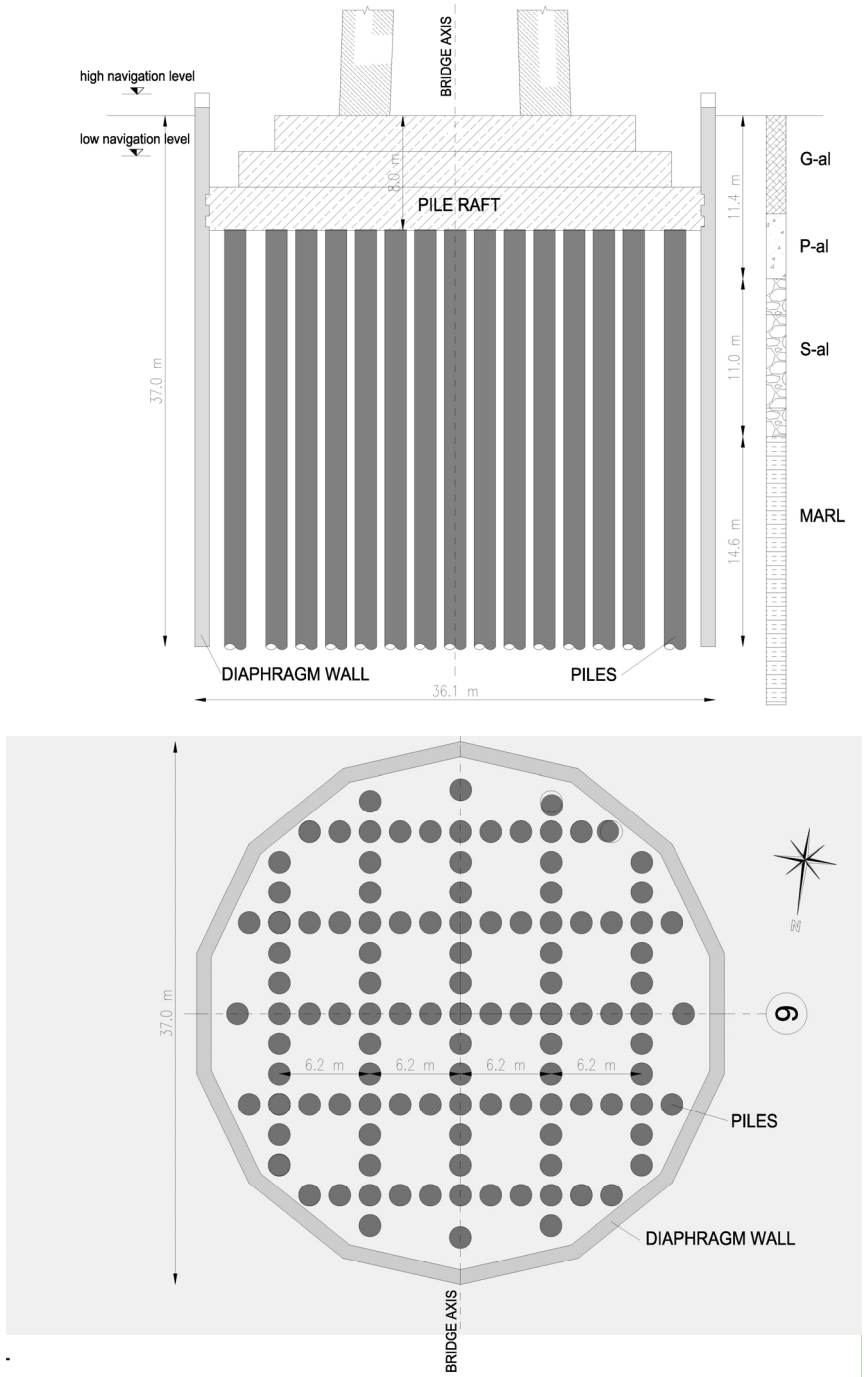

Fig. 19. Foundation layout at pier 6 (pylon foundation).

Fig. 20. Construction of pile cap at pier 6 (pylon foundation). Area outside the diaphragm wall is shown [14]

5.4.5 Trial piles

In order to verify soil parameters used for calculation and design of pile foundations of Sava-Bridge four trial piles at piers 5, 6 and 7 have been installed and tested. The test load was assessed as the work load multiplied by a safety factor of 2.125 according to EC 7-1 (design situation BS1).

As an example Fig. 21 shows measurement results of trial pile No. 2, situated at pier No. 6, with a diameter of 1.5 m, a length of 38.1 m and a maximum test load of 9.6 MN (considering a work load of 4.5 MN). Up to a depth of 8 m, which correspond to depth of designed pile raft, an elimination pipe was installed in order to avoid skin friction within the upper soil layers. At the maximum load stage measured settlements belong to 7.3 mm, with a plastic portion of 2.7 mm. The ratio of settlement to pile diameter is s/D = 0.0049 related to total settlements [17].

The measurement results have shown that a base resistance of q_b = 122 kN/m² (equates to 216 kN) occurred at the maximum load stage. The average skin friction within layers P-al, S-al and Marl is approx. q_s = 56 kN/m².

By means of measurement results (particularly with regard to measured deformation) it can be assumed that the ultimate limit state was not reached with the test load of 9.6 MN. This means, that the bearing capacity must be higher than the test load. Hence testing results are better than expected. Due to rock-similar behaviour of marls skin friction within layers P-al and S-al (granular soil) was not mobilised, because of low vertical deformations.

Although the ultimate bearing capacity of base resistance and skin friction of each single layer could not be determined by pile load test, it was shown that the intended work loads can be transferred to the ground with low deformations and sufficient safety. Thus design parameters of Geotechnical Interpretive Report have been approved and can be used for designing the pile foundations [18], [19].

Finally, the results of these trial piles provide also information about the settlements of a single pile and are used for settlement calculations.

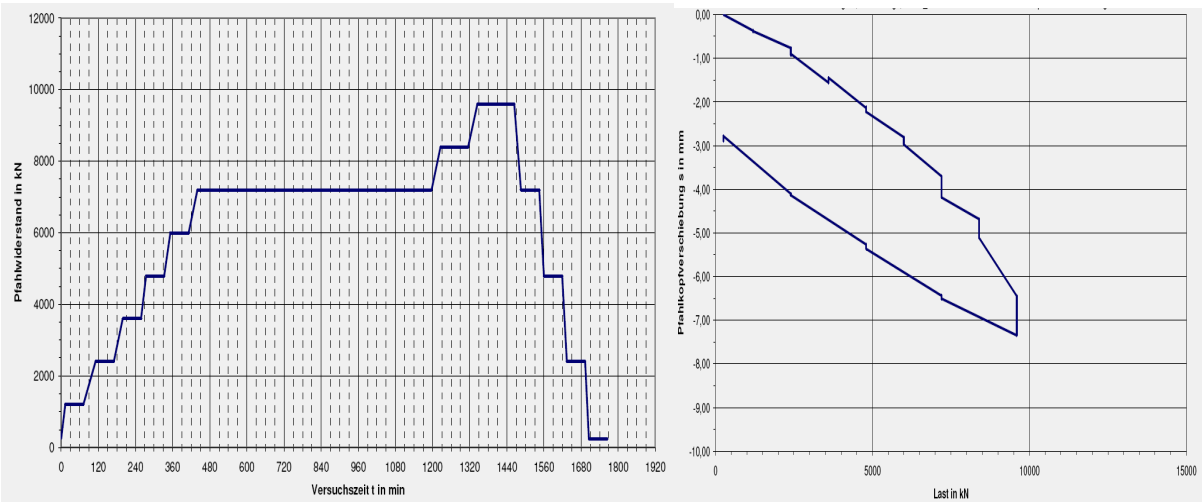

Fig. 21. Results of pile load test performed at pier No. 6. [17]

5.4.6 Settlement calculations

In the static model of the superstructure a stiffness matrix is integrated at each pier in order to consider the pile foundation. Furthermore, additional differential settlement Δs with ±1.0 cm ($|\Delta s|$ = 2.0 cm) is considered at each support using the elastic stiffness of the structure, whereas at the pylon axis a settlement Δs of 5.0 cm and a rotation $\Delta\varphi$ of ±1.0‰ are considered. Due to ground conditions at piers 7 and 8 (rock) and the results of trial piles differential settlement Δs with ± 0.5 cm ($|\Delta s|$ = 1.0 cm) is assumed for the axis 7 and 8.

In order to check the assumptions in the static model of the superstructure settlement calculations have been performed. The foundation of the piers is within rock-like marl at piers 1 to 6 and within sandstone at piers 7 and 8. Therefore, the piles predominantly work as end bearing piles. The total settlements (s_{total}l) of the foundation consist of the settlements of the single piles (s_1) as well as the settlements of the pile group (s_2): $s_{total} = s_1 + s_2$ (see DIN 1054)

The settlements of a single pile are derived from pile load tests (trial piles) taking into account the maximum working load of a single pile. For settlement calculations of the pile group the envelope area of all piles (resp. diaphragm walls at pier 6) in the foundation depth is taken as a basis. For this quasi-monolith the settlement calculations are performed like a shallow foundation with a deep foundation level according to DIN 4019. The total load is distributed over the total base of this quasi-monolith. The overburden stress in the foundation depth is compared with the additional stress due to bridge loads. The thickness of the compressible layer is limited to a depth (limit depth), where the additional stress reaches 20% of the overburden stress.

Settlement calculations have to be performed with working loads without safety factors. Therefore, separate static calculations without any safety factor have been carried out in order to calculate working loads of each pile.

For an example, at pier 6 (pylon axis) an average stress of 581 kN/m² occurs in the foundation depth. The additional stress of 39.6 kN/m² due to dead load of concrete elements (piles, diaphragm wall, pile cap) has to be added, so that the total additional stress for settlement calculation amounts to 620 kN/m².

Depending on the modulus of compression for the marl and the limestone (beneath marl) the settlements of foundation are estimated to

$$s_2 = 3.0 \text{ cm to } 4.0 \text{ cm}$$

with a limit depth of 70.3 m. With an estimated settlement of the single pile

$$s_1 = 0.5 \text{ cm}$$

the total settlements for final stage amount to

$$s_{total} = s_1 + s_2 = 3.5 \text{ cm to } 4.5 \text{ cm}$$

Differential settlements of the pier foundation due to load distribution within the pile group can be derived from settlement calculations of the box-shaped foundation and are estimated with $\Delta s \leq 0.5$ cm.

Regarding the rotation of the pier foundation several calculations have been performed in order to consider load distribution, inclined rock surfaces in longitudinal and transversal directions and varying soil /rock modulus. Due to the results it is recommended to set a rotation of 1‰ for the pier foundation in the static model.

Table 8. Estimated settlements at final stage [20]

Pier	Estimated settlements at final stage
Pier 1	1.5 – 2.5 cm
Pier 2	4.0 – 5.0 cm
Pier 3	3.5 – 4.5 cm
Pier 4	2.5 – 3.5 cm
Pier 5	2.0 – 3.0 cm
Pier 6	3.5 – 4.5 cm
Pier 7	0.0 – 1.0 cm
Pier 8	0.0 – 1.0 cm

The settlement calculations for final stage (see Table 8 above) have shown that the settlements of pier 1 are less than the settlements of the other piers, because of lower load from superstructure at this pier. The same conditions occur at pier 5: due to the cables a part of the load from superstructure is transferred to pier 6, which results in a minor foundation load at pier 5.

Piers 1 to 6 are founded within marl. However, at pier 6 the settlements are relatively low, although the total load is high in comparison to the other piers. The reason for that is that at pier 6 the limestone (rock) is only a few meters below foundation level. This limestone is considered only at pier 6 in the settlement calculations, because the interface was explored only in exploratory drillings at pier 6.

However, for the superstructure the differential settlements due to Table 8 are not relevant, because construction stages and settlements, which have already occurred until bearings for superstructure are installed, are not considered. Therefore, additional settlement calculations for the following construction stages have been performed:

- Construction stage A: Start of construction works
- Construction stage B: Completion of pier construction
- Construction stage C: Launching of superstructure is finished
- Final stage

Only the settlements respectively differential settlements which occur after construction stage C have an influence on the superstructure, because bridge bearings are installed just before this construction stage. However, these calculations have shown that the assumptions for the static model of superstructure are on the safe side and that the expected differential settlements are lower than those considered in the static calculation of the superstructure.

6. Summary and conclusions

In conclusion it is pointed out that all foundations of Danube river bridges in Austria with bored piles or box-shaped pile groups, built during the last decades were designed in such a way that they have both sufficiently high global safety against failure and also sufficiently high safety concerning the behaviour during life time or in the wording of Eurocode 7 Part 1 in the serviceability limit state. All these bridges show more than satisfactory settlement behaviour.
The first application of Eurocode 7 Part 1 in connection with our national application rules for Austria is not for a new Danube river bridge in Austria but for the preliminary design of a project of a new bridge abroad crossing another large river in Europe, the Sava in Belgrade near its mouth into the Danube.
Concerning the principles of the design concept for pile foundations according to Eurocode 7 Part 1 the authors of this contribution want to point out the following thoughts and ideas:

- Eurocode 7 Part 1 encourages designers by its correlation factors ξ_1 and ξ_2 to perform more pile load tests and if so to perform not only one test, as these factors are 1,4 for only one test (n = 1) with a limit value of 1,0 for $n \geq 5$. Nevertheless it will be possible in Austria to perform pile load tests on large diameter bored piles only in such exceptional cases when very unclear ground conditions were found or because of economic reasons when a high number of piles can be saved due to favourable test results.
- In the case of bridge piers in the river bed of Danube or rivers like Danube pile load tests in situ will be hardly possible. Therefore the pile foundation design for such bridge piers will be based on data given in tables in our national standard and based on long term experience also in future.
- For special solutions like box-shaped pile group foundations the design rules of Eurocode 7 Part 1 can only be understood as principle guidelines and in addition national determinations will be necessary for the future based on the previous design practice and the affiliated experience. Proposals for this additional design rules to be incorporated in ÖNORM B 1997-1-3 were already drafted.

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